Preliminary Risk-Based Flood Damage Analysis
Green River Flood Control Zone District
King County, Washington

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EXECUTIVE SUMMARY

Shannon & Wilson, Inc. has completed a preliminary risk-based analysis for the King County Department of Natural Resources and Parks (DNRP) to determine potential flooding-related damage that could occur in the Green River Flood Control Zone District (GRFCZD) as a result of levee and revetment instability. The objective of the study was to develop a preliminary estimate of the expected annual damage to structures and structure contents within the GRFCZD floodplain. This type of risk-based analysis is a method of performing studies in which uncertainty in technical data is taken into account. Complex systems such as rivers, levees, and flood plains, and their economic and other human uses, have many uncertainties associated with their evaluation. Risk-based analysis allows engineers, scientists, planners, and community leaders to identify these uncertainties and to quantify their effect on decision-making processes. The basic principles of risk-based flood analysis are similar to a traditional, deterministic approach in that both approaches use analytical tools and engineering methodologies founded on sound, scientific evidence and experienced, professional judgment. However, in the risk-based approach, the uncertainties associated with the analysis and design are quantified and included in the presentation of the results and design recommendations.

Our analysis utilized the HEC-FDA, Flood Damage Reduction Analysis software developed by the U.S. Army Corps of Engineers. This analysis tool considers river discharge probabilities, river stage-discharge functions, probability of failure of flood damage reduction structures (levees and revetments), and probable damage to structures, and other property and goods to obtain an estimate of expected annual damage. The approach taken for this preliminary risk-based analysis of the GRFCZD was to adopt a simplified model of the river, levees, and economic impacts of flooding. While this approach imposes significant uncertainty on the result, it does provide a mechanism for estimating the order of magnitude of the economic impacts of flooding in the GRFCZD and for identifying the most significant variables for future, in-depth analysis.

The river is represented in the analysis by a discharge-probability function and a river stage-discharge function. A discharge-probability function relates annual probability of exceedance to flow rate. Annual probability of exceedance is the probability that a given flow rate will be exceeded in any year. A stage-discharge function relates water elevation (stage) to flow rate of the river. For this preliminary study, we used probability-discharge and stage-discharge curves obtained from single locations to represent the entire reach of the GRFCZD.

A river stage versus levee failure probability function expresses the relationship between probability of levee (or bank) failure and river stage. The modes of failure considered in our analyses include under-seepage, through-seepage, slope stability of the levee on the riverside under static conditions, slope stability of the levee on the landside under static conditions, slope stability of the levee on the riverside during rapid drawdown, and scour (erosion) due to river flow. Each of these failure modes was evaluated separately, and then combined using probabilistic methods to determine a composite river stage versus levee failure probability function. Evaluating each failure mode independently also allows determination of the failure mode that most significantly contributes to the probability of failure at any given river stage.

A stage-damage function expresses the relationship between water level (stage) to the dollar cost of damage incurred. Detailed knowledge of the land use, structure type and value, content value or content value as a percent of structure value, elevation of structure first floor, percent damage versus stage, and other details for each river reach are needed to define the stage-damage function. Because it was not feasible to obtain this information given the limited scope for the preliminary study, historical flood damage data was used to establish the stage-damage function.

After the four aforementioned functions were defined, they were input into the HEC-FDA software and combined to estimate the expected annual damage due to flooding. The output of the program indicates that the estimated damage per year for the GRFCZD under existing conditions is $65,730,000 with a standard deviation equal to $330,000. This is a preliminary estimate based on the generalized conditions that were selected for evaluation due to the limited time frame and budget. This estimate of damage per year could be more accurately defined given a larger scope including further investigation and exploration of the river, levees, floodplain characteristics, and adjacent land use. The recommendations for further work are also outlined in this report.

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ABBREVIATIONS AND ACRONYMS

cfs	cubic feet per second
COV	coefficient of variation
DNRP	Department of Natural Resources and Parks
FEMA	Federal Emergency Management Agency
FS	factor of safety
GRFCZD	Green River Flood Control Zone District
HAH Dam	Howard A. Hanson Dam
NOAA	National Oceanic and Atmospheric Administration
P_f	probability of failure
R	reliability
RM	River Mile
USACE	U.S. Army Corps of Engineers
USDA	U.S. Department of Agriculture
USGS	U.S. Geologic Survey
WRIA	Water Resource Inventory Area

PRELIMINARY RISK-BASED FLOOD DAMAGE ANALYSIS
GREEN RIVER FLOOD CONTROL ZONE
KING COUNTY, WASHINGTON

1.0   INTRODUCTION AND STATEMENT OF WORK

This report presents Shannon & Wilson, Inc.'s preliminary risk-based analysis of potential flooding-related damage that could occur in the Green River Flood Control Zone District (GRFCZD) as a result of levee and revetment instability. The GRFCZD is a cooperative operation of King County and the municipalities of Auburn, Kent, Renton, and Tukwila. The study was authorized by King County Department of Natural Resources and Parks (DNRP) as Project No. 089518, Work Order No. T00839X, Contract No. T00839T. The objective of the study was to develop a preliminary estimate of the expected annual damage within the GRFCZD considering river discharge probabilities; river stage-discharge functions; probability of failure of flood damage reduction structures (levees and revetments); and probable damage to land, structures, and other property and goods. The results are intended to provide the GRFCZD with an initial basis for developing a long-term levee- and revetment-maintenance and repair funding plan and to identify the elements of the risk-based analysis that may require a more thorough evaluation.

2.0   HISTORY OF FLOODING AND FLOOD CONTROL ALONG THE GREEN RIVER

The headwaters of the Green River are at River Mile 91 in the Cascade Mountains and the river mouth is at Elliot Bay (RM 0.0). The total size of the Green River drainage basin (a part of Water Resource Inventory Area No. 09.0001 or WRIA 9) is approximately 483 square miles. Precipitation and runoff from approximately 220 square miles of the basin are controlled by the U.S. Army Corps of Engineers (USACE) Howard A. Hanson Dam (HAH Dam) at RM 64.5, which went into operation in December 1961.

The GRFCZD encompasses approximately 30 miles of the Green River in south King County, extending from River Mile (RM) 6.5 near the junction of Highways 99 and 599 to RM 36.8 at the confluence of Big Soos Creek and the Green River near SR 18 (Figure 1). The GRFCZD is generally about 5 miles wide and includes substantial portions of the cities of Auburn, Kent, Renton, and Tukwila, and smaller portions of Federal Way, Seatac, and Des Moines.

The Lower Green River Sub-Watershed, which includes the majority of the GRFCZD, is highly urbanized with areas of dense commercial and industrial development (King County Department of Natural Resources and Parks, DNRP 1988). Land use in 1988 was estimated at 60 percent urban; 30 percent non-agricultural rural; and 10 percent agricultural, forests, and parks. The population of the cooperating cities in the GRFCZD has grown by from about 175,000 in 1990 to nearly 250,000 in 2000 (U.S. Census 2000). Further urbanization of the rural areas is expected in the future, consistent with local comprehensive plans.

Prior to construction of HAH Dam, flooding along the Green River was reported to be an "almost annual" event and "floodwaters … periodically spread unimpeded across the Green River Valley" (USACE, 2001 and Seattle Public Utilities, 2000). The USACE reports that the Green River has flooded more than 30 times in the past 70 years. During the last major flood in 1959 (prior to the construction of HAH Dam) floodwaters reportedly reached the second floors of houses in the valley (USACE, 2001).

Since the construction of HAH Dam, flooding in the Green River Valley has continued to be a regular but controlled event. Water is held at HAH Dam during periods of high precipitation or runoff, but is released downstream as quickly as possible to restore reservoir storage capacity in anticipation of the next storm. In addition, incremental channelization and bank-protection projects (levees and revetments) that continued through the 1960s and 1970s brought the river to its current, controlled state (DNRP, 2001).

Flood phases in the Green River Valley are defined by flows at the Auburn gage located at RM 31.3 (DNRP, 2001). Table 1 shows Green River flood phases, and recent flood events are listed in Table 2. Several of the recent flooding events within King County were declared flood disasters by the federal government. The Federal Emergency Management Agency (FEMA) estimates that approximately 140,000 acres in King County are at risk of flooding during a 100 year flood event (Office of Emergency Management, 1998 and DNRP, 1997).

The Green River levees and revetments were built primarily in the 1960s and 1970s and were funded by bond issues that expired in the early 1980s. A flood control study was completed by the USACE in 1983; however, the USACE declined to participate in levee improvements at that time (DNRP, 1997). Periodic, site-specific, levee and revetment maintenance and repairs have occurred since that time under the sponsorship of King County. Funding for the GRFCZD's major levee and revetment projects is generated by and ad valorem- or "value added"- tax levy on all properties with the GRFCZD boundaries. Over $750,000 in tax levy revenue is collected

annually in the GRFCZD with approximately one-half of the revenue allocated to funding major levee and revetment projects. The USACE and FEMA occasionally contribute funding for major levee and revetment projects.

3.0   RISK-BASED ANALYSIS

Complex systems such as rivers, levees, and flood plains, and their economic and other human uses, have many uncertainties associated with their evaluation. Risk-based analysis allows engineers, scientists, planners, and community leaders to identify these uncertainties and to quantify their effect on decision-making processes. The basic principles of risk-based flood analysis are similar to a traditional, deterministic approach in that both approaches use analytical tools and engineering methodologies founded on sound, scientific evidence and experienced, professional judgement. However, in the risk-based approach, the uncertainties associated with the analysis and design are quantified and included in the presentation of the results and design recommendations.

The variability and uncertainty in a risk-based analysis of a flood control project arise from several sources. They include natural variability of hydrologic events and river hydraulics, imperfect measurement of natural conditions and phenomena, uncertainties in making simplifying assumptions, uncertainties introduced by using simplified models to represent complex processes, and uncertainties in measuring and predicting social and economic impacts. The traditional approach accounts for these uncertainties by applying safety factors, considering worst-case scenarios and using conservative design methods. With advances in hydrology and the use of statistics, and the widespread availability of powerful desktop computers, it is now feasible to qualitatively and quantitatively express the uncertainty associated with the analysis and design process. The result is a more well defined engineering recommendation and a more informed decision-making process.

4.0   APPROACH AND METHODOLOGY

4.1   Overview

The approach taken for this preliminary risk-based analysis of the GRFCZD was to adopt a simplified model of the river, levees, and economic impacts of flooding. Given the limited schedule and funding available for the preliminary study, the analysis approximates the Green River within the GRFCZD as a uniform reach, an assumption applied to river hydrologic and

hydraulic characteristics, levees and levee performance, and economic impacts. While this approach imposes significant uncertainty on the results, it does provide a mechanism for estimating the order of magnitude of the economic impacts of flooding in the GRFCZD and for identifying the most significant variables for future, in-depth analysis.

4.2   HEC-FDA Model

Computations for the preliminary risk-based analysis of flooding in the GRFCZD were completed using a USACE software program for risk-based analysis of flood control projects called HEC-FDA, Flood Damage Reduction Analysis (USACE, 1998). The purpose of HEC-FDA is to perform an integrated risk-based hydrologic engineering and economic analysis. Typically, HEC-FDA is used to compare flood damage reduction alternatives to a no-action alternative in order to determine the most economically viable alternative. For this project, however, HEC-FDA was used to estimate the potential economic damage associated with breach of flood damage reduction structures under existing river conditions.

The primary inputs required to perform an HEC-FDA analysis are one or more river discharge-exceedance probability functions, stage-discharge functions, stage-levee failure probability functions, and stage-damage functions (the HEC-FDA input functions are described in more detail in Sections 4.3 through 4.6 below). Different levels of uncertainty may be assigned to each of these functions. The HEC-FDA program uses a Monte Carlo simulation technique to compute an expected value of flood damage. The Monte Carlo method is a statistical technique that uses the input probability and function uncertainties to calculate a value of flood damage for many possible scenarios. The results of all scenarios are then averaged and a standard deviation is calculated to yield the expected value of flood damage and an associated uncertainty in the answer.

4.3   Discharge-Probability Function

A discharge-probability function relates annual probability of exceedance to flow rate. Annual probability of exceedance is the probability that a given flow rate will be exceeded in any year. Consider the following hypothetical example, a flow rate of 10,000 cfs may have an exceedance probability of 0.5, which implies that, on the average, there is a 50-50 chance that flow will exceed 10,000 cfs in any year. The discharge-probability function is often expressed in terms of return period, which is the inverse of the probability. Thus, to continue the hypothetical example, an annual exceedance probability of 0.5 is equal to a return period of two years; i.e., a flow in excess of 10,000 cfs would be expected to occur every two years, on average.

A discharge-probability function for the preliminary analysis of the GRFCZD was derived from peak flow measurements obtained at USGS Station Number 12113000 on the Green River near Auburn (near the upstream limit of the GRFCZD). Annual peak flow measurements for the period of 1937 to 1999 from the Auburn gage are presented on Figure 3. An examination of peak flow measurements at USGS Station Number 12113500 on the Green River at Tukwila (period of record 1961 to 1984) shows that the peak flows at this gage are very similar to peak flows at the Auburn gage. This indicates that the Green River in the GRFCZD generally appears to respond in a uniform manner to peak flow conditions. A correlation plot showing peak flows at the Tukwila gage versus peak flows at the Auburn gage is presented on Figure 3a.

In developing the discharge-probability function for the Auburn gage, it is necessary to differentiate data before and after the completion of HAH Dam. Prior to the construction of HAH Dam, the Green River was unregulated, and annual peak flow rates in excess of 28,000 cfs were recorded at the Auburn gage. Since the completion of HAH Dam, the River is regulated to limit the maximum flow rate at the Auburn gage to about 12,000 cfs. The data in Figure 3 indicates that operations at HAH Dam have largely limited peak flows at the Auburn gage to the target flow rate; the maximum flow rate recorded at the Auburn gage was 12,400 cfs in February 1996. For this project, it is appropriate to use only the flow data for the period after the construction of HAH Dam under the assumption that future flow rates in the Green River will continue to be controlled by operations at HAH Dam. Furthermore, it was assumed that the dam would be properly maintained, and the possibility of a complete dam failure was not considered. Consequently, the peak discharge was limited to a maximum of 12,000 cfs. However, uncertainties associated with the discharge-probability curve permit flow rates in excess of 12,000 cfs to be considered in this analysis.

Discharge-probability functions for the Auburn gage are presented on Figure 4, which includes a discharge-probability curve for the data prior to completion of HAH Dam (Pre-Dam) and for the data after completion of the dam (Post-Dam). The Post-Dam curve is also bracketed by the 90 percent confidence interval curves (5 percent and 95 percent). The Pre-Dam curve was derived from 25 peak annual flow measurements and the Post-Dam curve was derived from 40 peak annual flow measurements. USGS Water Resources Council methods were used to derive these curves. The uncertainty associated with the Post-Dam discharge-probability curve is a function of the variability of the observed peak annual flows. For reference, the same data that was used to prepare Figure 4 is also presented on Figure 5 as return period versus discharge.

4.4   River Stage-Discharge Function

Stage is the elevation of water in the river and discharge is the flow rate of water. Thus, a stage-discharge function relates water elevation to flow rate. In general, the stage-discharge curve at each sub-reach of a river will depend on the hydrologic and hydraulic properties of the river reach.

For this preliminary study, we have assumed that the stage-discharge curve developed for the Midway (RM 19.5) reach is representative of conditions the Green River through most of the GRFCZD. The stage-discharge curve for the Midway reach is shown on Figure 2. The range of stages at the Midway location is approximately 22 feet. The maximum discharge is well above the target discharge of 12,000 cubic feet per second (cfs) for Green River flood control, as defined in the Congressional authorization for operating HAH dam. The target stage for flows not to exceed 12,000 cfs at the Midway location is approximately 35 feet. However, our analysis considers the possibility of rare events that could exceed the target discharge and stage.

The uncertainty associated with the Midway discharge vs. stage curve is apparently not documented. Therefore, estimates of the stage uncertainty for ungaged river reaches as recommended by the USACE were applied (USACE, 1996). The total uncertainty, which is expressed as a standard deviation of stage, includes a component of natural variability and a component of computational error arising from calculation of a stage-discharge curve. The USACE stage uncertainty recommendations are based on stage data from a number of other rivers. The best current estimate of the standard deviation of stage is 2.5 feet, as shown on Figure 2. This standard deviation of stage should be evaluated in a future study phase to determine if the estimate is reasonable and consistent with field observations.

The USACE definition of flood stage and the target discharge from operations at HAH Dam are also shown on Figure 2 to give some perspective to the stage-discharge relationship and to indicate the range of stages that are beyond the flood control target ranges.

4.5   Levee Failure Probability Functions

A levee failure probability function expresses the relationship between probability of levee (or bank) failure and river stage. Each of the possible failure modes is evaluated separately and then combined to obtain the composite levee failure probability function. This technique also allows determination of the failure mode that most significantly contributes to the probability of failure and may aid in prioritizing projects considered in a levee repair and rehabilitation program. The

methodology as described in the following sections is derived from several papers (Shannon & Wilson and Wolff, 1994 and Wolff, 1989 and Duncan, 2000 and Duncan et al., 1999).

Traditionally, a tri-linear curve was used to represent the stage-probability of failure relationship (USACE 1996). However, recent research (Duncan, 2000 and USACE, 1999) presented simple techniques combining traditional engineering analyses with probabilistic methods to obtain a more detailed relationship. Using these techniques, the stability at several intermediate river stages can be evaluated to define a complete curve showing river stage versus probability of levee failure.

An important step in determining a levee failure probability function is to define the levee geometry and soil parameters to be used in the analyses. Because of the limited scope and budget and due to the substantial uniformity of the 1960's era levee geometry present, a generalized levee cross section (Figure 6) was used to represent the entire length of the river within the GRFCZD. Furthermore, it was assumed that the same protection is offered by the entire levee system, and therefore the freeboard during a given river stage is the same at each location along the alignment. Visual assessments by King County staff during peak flow periods in 1999 have confirmed the general validity of this simplifying assumption. In a more detailed future study phase, the river could be divided into multiple reaches to more accurately define problem areas. The levee configuration and range of soil parameters used here are based on previous GRFCZD studies completed by Shannon & Wilson, Inc. (Shannon & Wilson, Inc., 1995 and 1999), as well as soil subsurface information obtained from the USACE and estimates of river and levee geometry from King County. These parameters are defined as the expected values, or mean values, used in the stability analysis.

Unlike traditional analyses, probabilistic stability analyses allow a range of soil parameters and levee geometry to be considered. This range is typically defined as one standard deviation above and below the expected (mean) value. There are several accepted methods for determining the range of values. If a large amount of data is available from laboratory tests, field tests, or field reconnaissance, the mean and standard deviation values for each parameter can be determined using statistical methods. Alternatively, when limited data is available, typical values of the coefficient of variation (COV) for each parameter can be obtained from the literature (Shannon & Wilson and Wolff, 1994 and USACE, 1996). COVs, in this case, are used to compute a standard deviation for each parameter or used as a guide to develop reasonable parameter ranges. Note that COV equals the standard deviation divided by the mean of a parameter. Given the limited scope and budget available for the preliminary analysis of the GRFCZD, parameter

ranges were determined by the alternative method using available data and COV values obtained from the literature. This approach introduces greater uncertainty into the analysis and more refined estimates of parameter values and parameter variation should be obtained for each river reach in a future study phase.

For the stability analysis of the levees within the GRFCZD, several modes of failure were independently evaluated. The modes of failure considered in our analyses include under-seepage, through-seepage, slope stability of the levee on the riverside under static conditions, slope stability of the levee on the landside under static conditions, slope stability of the levee on the riverside during rapid drawdown, and scour (erosion) due to river flow. Other modes of failure exist, including seismic, man-induced failures (such as excavating at the toe of the landside slope), and animal-induced failures (burrow holes). The seismic failure mode could not be evaluated as part of this preliminary study, but should be included in more detailed future studies. The man- and animal-induced failure modes are considered to be less likely and are generally within the statistical error of the analyses employed.

The general method used to determine the levee failure probability function is as follows. First, using the expected values in the generalized cross section (Figure 6), the stability of the levee at a given river stage was evaluated considering one possible mode of failure. Second, one of the parameters was changed to its estimated low value (mean minus one standard deviation), and the stability was re-evaluated. Likewise, the parameter was changed to its estimated high value (mean plus one standard deviation), and the stability was re-evaluated. Each of the controlling parameters was subsequently independently changed to its low and high values and the stability evaluated. (By changing each of the parameters independently, it becomes apparent which parameter contributes most to the uncertainty of the stability.) Next, using statistical methods, the probability of failure was determined. The above steps were repeated for several different river stages to determine the probability of failure versus river stage for one independent mode of failure. Each mode of failure was similarly evaluated, and the individual functions were then combined to establish the composite levee failure probability function. The following sections briefly describe the methods and assumptions used to evaluate stability for each of the aforementioned modes of failure.

4.5.1 Under-seepage

The stability of the levee during under-seepage conditions is based on methods used by the USACE (USACE, 1956). Under-seepage is typically a concern when a pervious levee foundation material is overlain by a less pervious blanket material near the landside toe of the

levee. Under these conditions, the head beneath the landside levee toe could result in erosion of foundation material (piping) or a reduction in soil strength (quick conditions). Based on our previous experience in the area, this condition exists in the GRFCZD levees (for example, at locations involved in flood fighting along the Segale Levee at RM 15.5 in 1996). The parameters used in the evaluation are shown in Table 3. The stability was evaluated by determining the maximum exit gradient and comparing it to the limit state gradient of 0.85. Probabilistic methods were then used to calculate the probability of failure.

4.5.2   Static Slope Stability

The stability of the levee slopes under static conditions was evaluated using the PCSTABL5M slope stability program by Purdue University along with the STEDwin 2.17 pre- and post-processor by Annapolis Engineering Software (PCSTABL5M, 1988 and Van Aller, 1999). Because the slope is under static conditions, it is assumed that the river stage has remained at a given level for a sufficient period of time to allow the water within the soil to come to a steady state condition. Further, where the river is above the landside levee toe, it is assumed that the piezometric line (water table) within the levee is a straight line from the water stage level on the riverside to the landside toe of the levee.

Several other assumptions were made for the static slope stability analysis. First, limitations were placed on the location of the failure surface. In riverside cases, the failure surface was required to pass through or below the toe of the slope. This assumption prevented shallow failures, also known as surface raveling, from controlling the behavior of the slope and giving misleading low factors of safety. Furthermore, for the landside, only failure surfaces that caused a reduction in the freeboard of the levee were considered. In other words, shallow failure surfaces were not critical because they would not reduce the freeboard of the levee system. (Based on King County's record of regular visual assessment and maintenance of the levees, it was assumed that any shallow failures would be repaired before they would affect the integrity of the levees under flood conditions.) Also, planes of weakness were assumed not to exist within the soil, and therefore only circular failure surfaces were considered. The parameters employed for static slope stability analyses are presented in Table 4. The slope was analyzed by finding a factor of safety (FS) given the assumptions and parameters listed above. The resulting FS was compared to the limit state (FS = 1.0) and the probability of levee failure was calculated.

4.5.3   Rapid Drawdown Slope Stability

For the rapid drawdown analysis, it was assumed that the river stage remained at a given level for sufficient time to establish steady state conditions within the soil. Then, the river level was dropped to a level 20 feet below the crest of the levee. For ease of calculation, the rapid drawdown was assumed to be instantaneous. This implies that water is not allowed to drain from the soil and that pore pressures in the soil remain at the pre-drawdown level. Because this only affects stability on the riverside of the levee, the landside was not evaluated for this case. Stability charts from were used to calculate the FS of the riverside slope during rapid drawdown conditions (Morgenstern, 1963). The controlling parameters employed in the rapid drawdown analysis were identical to those used in the static analysis and are listed in Table 4.

It should be noted that the dashed line plot in Figure 7 is the probability of failure if rapid drawdown has the same probability of occurrence as the other modes of failure. However, in comparing the assumption of complete saturation before drawdown and the assumption of instantaneous drawdown resulting from operation of HAH Dam, it would appear to be less likely that rapid drawdown will occur during a given river stage. The USACE generally attempts to limit stage drawdown from operations at HAH Dam to less than one foot per day and is only authorized to proceed at one foot per hour at most. Stage drawdown of one foot or less per day would not generally create a rapid drawdown condition; however, stage drawdown of one foot per hour could create a rapid drawdown condition. For the purpose of this study, we therefore have conservatively assumed that a rapid drawdown condition has a 25 percent probability of occurrence at any river stage, and the resulting curve is shown as the solid line in Figure 7. The 25 percent probability factor implies that every four years, on average, HAH Dam operations would create a rapid drawdown condition in the GRFCZD levees.

4.5.4   Scour and Erosion

Scour and erosion occur within riverbeds because the forces exerted by the flowing water are greater than the resisting forces of the riverbed material. The resisting forces are mostly derived from the weight of the individual particles of soil or rock and are dependent on the grain sizes of the riverbed material. The forces exerted by the flowing water can be estimated using the slope of the river and the depth of flow. Several studies have resulted in relationships between forces exerted by the flowing water and median grain size of the river channel material (Vanoni, 1977 and Washington Forest Practices Board, 1997). Using such relationships, scour would be expected to occur along most of the toe of the riverside slope.

There are several other factors that contribute to the uncertainty of the scour analysis. First, the river channel consists of straight stretches and bends throughout the GRFCZD. As the water flows through a bend, the velocity, and therefore exerted force, of the water along the outside of the bend is greater and is more likely to cause scour. However, the bends were not considered, and it was assumed that the estimated water forces as described above represent the overall average in the GRFCZD. Second, stretches of the riverbank and/or river channel are protected by riprap. It was assumed that approximately 50 percent of the river is protected by riprap that is properly sized to prevent scour and erosion. Finally, even if scour occurs, it does not necessarily lead to failure of the levee. Instability would only occur if enough material were removed from the toe of the levee to cause instability in the overall slope. Based on King County's record of regular visual assessment and maintenance of the levees, it was assumed that this phenomenon only occurs about 10 percent of the time in which scour is initiated. The resulting probability of failure function for erosion/scour is presented in Figure 7.

4.5.5   Other Possible Modes of Failure

As suggested previously, other possible modes of failure are non-quantifiable and/or need further research to develop analytical techniques. These modes of failure include seismic failures, areas of weakness due to animal burrowing, unsatisfactory construction resulting in planes of weakness, vandalism, and activities adjacent to the levee that could compromise its stability during flooding.

4.5.6   Composite Levee Failure Probability Function

Following the determination of the levee failure probability functions for each of the modes of failure, they were combined to form the composite levee failure probability function to be used in the HEC-FDA model. The composite probability of failure (P_f) is obtained by multiplying the reliability functions (R) from each mode of failure. The reliability function is defined as $R_i = 1 - P_{f_i}$. The composite, levee failure probability function is shown as the bold line in Figure 7. The simplifying assumptions stated above for determination of the levee failure probability function are generally conservative from the perspective of not overstating the likelihood of levee failure. Therefore, the results can be taken as a reasonable lower bound for the composite risk of failure given the level of assumed parameter uncertainties.

4.6   Stage-Damage Functions

A stage-damage function expresses the relationship between depth of water in a river (stage) to the dollar cost of damage incurred (note that a typical stage-damage function will have zero damage until the depth of water in the river exceeds the bank height of the river). This aspect of risk-based analysis of flooding is the most difficult part to evaluate (NOAA, 2000 and Pielke and Downton, 2000). The quality of flood damage data varies considerably from year to year and from location to location. In general, flood losses are reported at the national and state levels with some county level reporting. For example, it has been estimated that $15 million in damage was done in King County as a result of the floods of November 1990 to March 1991 (King County, 1998). FEMA estimates that about 140,000 acres of King County are at risk from a 100-year flood and that 4,000 to 18,000 homes are vulnerable (USACE, 1996).

The HEC-FDA program requires that a stage-damage function be entered directly or that detailed economic impact data be entered so that the program can calculate a stage-damage function. The required economic impact data includes structure type and value, content value or content value as a percent of structure value, elevation of structure first floor, percent damage versus stage, and other details for each river reach that are beyond the level of effort available for this preliminary risk-based analysis.

Our approach for the preliminary analysis was to develop separate stage-damage functions for residential and non-residential properties based on observed river stages and historical damage estimates collected by the USACE Institute for Water Resources (Davis et al., 2000). The USACE residential stage-damage relationships are presented for several different structure types as: (1) stage versus percent of structure value damage and (2) stage versus content value damage as a percent of structure value. An aggregate residential stage-damage percentage relationship was computed for the preliminary analysis by averaging the USACE curves for the different structure types. The average stage-damage percentage relationship was used for all residential structures in the GRFCZD floodplain. Non-residential stage-damage percentage relationships were developed by reducing the residential curves by approximately one-third based on reported relative damage incurred for residential and non-residential structures (U.S. Department of Agriculture, USDA 1978).

The USACE stage-damage percentage curves were converted to stage-damage relationships for the GRFCZD using the estimated values of properties at risk as provided by King County. The King County data indicates that over 4,000 parcels are located within the 100-year flood plain in the GRFCZD. The total assessed value of improvements to these properties is estimated to be

$3,490,150,000. The properties include residential, commercial, industrial, agricultural, and public lands with an assessed land value of $1,946,389,380. Note that the assessed land value was not used in the computation of expected annual damage even though there could be some economic loss to raw land from major flood events.

Table 5 shows the distribution of the total assessed value of improvements between residential and non-residential parcels. Residential and non-residential stage-damage relationships for the GRFCZD were computed by multiplying the stage-damage percentages for structure and content times the total assessed improvement value and summing the results. The residential and non-residential composite stage-damage functions are presented on Figures 8 and 9, respectively, along with estimated 90 percent confidence intervals. The uncertainty associated with the stage-damage functions was also determined from the USACE stage-damage percentage curves.

It should be noted that the stage-damage curves used in the preliminary analysis of the GRFCZD do not include costs associated with damage to public infrastructure (such as roads, bridges, and water and sewer systems), agricultural property/structures/livestock, or costs of flood warning and flood fighting. The costs of personal injury, loss of life, lost wages, lost tax revenues, and other non-property flood damages were also not considered.

4.7   Model Performance and Output

The stage-discharge, discharge-exceedance probability, levee failure probability, and stage-damage functions described above were entered into the HEC-FDA program to calculate an expected annual damage from flooding in the GRFCZD. To evaluate the sensitivity of the HEC-FDA model to variations of input, several runs were made with different levee failure probability functions and different uncertainties for the stage-discharge function. Results of the HEC-FDA calculations and sensitivity evaluation are presented in Section 5.0 and are summarized on Table 6.

5.0   RESULTS

The output of the HEC-FDA program indicates that the expected annual damage for the GRFCZD using conservative assumptions under existing conditions is $65,730,000 with a standard deviation equal to $330,000. The expected annual damage to residential structures and contents is approximately $3,730,000 and to non-residential structures and contents is $62,000,000. The expected annual residential damage represents about 3.4% of the estimated total assessed value of improvements and contents of the residential structures within the

GRFCZD floodplain. The expected annual non-residential damage is about 1.1% of the estimated total assessed value of improvements and contents of the non-residential structures within the GRFCZD floodplain.

The stage-discharge and discharge-probability relationships at the Auburn gage are well defined by long-term measurements; however, the measurement uncertainty for the stage-discharge relationship at the Midway reach location has been conservatively estimated. To evaluate the effect of the conservative estimate of measurement uncertainty of 2.5 feet, the HEC-FDA analysis was also run with a less conservative estimate of measurement uncertainty of 2.0 feet. Under this assumption (all other assumptions being the same), the expected annual damage was found to be $59,300,000. A reduction in uncertainty in the stage-discharge relationship would result in a lower expected annual damage because of the reduced likelihood of flood events that exceed the target discharge of 12,000 cfs at the Auburn gage. Further reduction in the uncertainty associated with the stage-discharge relationship would further reduce the expected annual damage estimate.

To evaluate the influence of probability of levee failure on expected annual damage, two additional HEC-FDA cases were run. The initial case described above has a probability of levee failure of approximately 0.35 for river stages from 1 to 10 feet below levee crest. The two additional cases were run with probabilities of 0.25 and 0.05, respectively, for river stages 1 to 10 feet below levee crest (all other assumptions unchanged). For the 0.25 probability case, the expected annual damage is $49,000,000 and for the 0.05 case is $15,500,000. As anticipated, the expected annual damage falls as levee reliability increases. However, since the levee reliability was unchanged at the highest river stages, the expected annual damage remains high even as the levee reliability approaches 1.0 for intermediate river stages. That is, the expected annual damage is dominated by the high-water events for all cases considered in this simplified analysis.

Variations of the stage-damage relationships were not considered in the preliminary analysis. Reliable, site-specific flood damage information or estimates are not readily available and only general percent damage functions were used. The computed expected annual damages can be assumed to vary in proportion to the assumed stage-damage curves entered into the HEC-FDA program. Further refinement of the stage-damage curves in future study phases will be essential for reducing the uncertainty in expected annual damage estimates.

The type of risk-based analysis presented in this report uses a methodology in which uncertainty in technical data is taken into account. It provides greater insight into the problem than classical deterministic methods. By examining the functions that are input into the HEC-FDA program, it

is apparent which hydrologic, hydraulic, economic, and geotechnical factors contribute the most to the resulting estimates of damage per year.

6.0   LIMITATIONS

This report was prepared for the exclusive use of the King County Department of Natural Resources and the Green River Valley cities participating in the GRFCZD. The analyses and conclusions contained in this report are based on site conditions as they presently exist. Because of the limited time frame and scope of work, detailed site exploration was not completed. Generalized conditions were assumed to exist throughout the GRFCZD and may not be representative of the entire stretch of river in question. Furthermore, rivers are complex and dynamic systems that are continually changing due to erosion, aggradation, and deposition, and this variability is beyond human control. Therefore, regardless of the actions taken by King County, routine inspection and maintenance should be completed.

Within the limitations of scope, schedule, and budget, the conclusions and recommendations presented in this report were prepared in accordance with generally accepted professional engineering and geological principles and practice in this area at the time this report was prepared. We make no warranty, either express or implied. Shannon & Wilson, Inc. has prepared the attachment "Important Information About Your Geotechnical Report" (Appendix) to assist you and others in understanding the use the limitations of our reports.

7.0   RECOMMENDATIONS FOR FURTHER STUDY

The study presented herein is preliminary in nature and is not intended as a final estimation of yearly damage along the GRFCZD. The reliability of the expected annual damage would be greatly increased with an expanded scope of work including the items listed below.

Conduct a detailed topographic survey of the levee, river, and floodplain within the GRFCZD, including visual reconnaissance and inspection, photogrammetry, topographic surveys, subsurface exploration, and soil testing.

1. Divide the GRFCZD into several independent reaches based on data obtained from the existing conditions survey.

2. The estimated standard deviation of peak stage should be evaluated to determine if the estimate is reasonable and consistent with field observations.

3. Determine breach-flow patterns and breach-damage relationships based on floodplain survey.

4. Summarize history of levee damages, levee repairs, and flood fighting within the GRFCZD.

5. Define more accurate ranges of stability parameters based on site conditions survey, including seismicity concerns in the levee stability analyses.

6. Include the effects of flood duration.

7. Evaluate through-seepage and under-seepage independently using more rigorous engineering analyses.

8. Determine and include the effects of levee length on probability of failure.

9. Evaluate the time and resources available to repair the levee or conduct flood fighting activities (i.e., sandbagging and pumping) to avoid damage along critical reaches of the GRFCZD levee system.

10. Consider costs associated with personal injury and/or loss of life, loss of use, damage to public infrastructure, and emergency response in the event of a levee failure.

11. Prioritize recommended repairs to existing flood protection facilities, based on risk and cost.

Each of these items would be used to define the various sub-reaches within the GRFCZD and to prepare separate, more accurate discharge-probability stage-discharge, stage-levee failure, and stage-damage curves for each sub-reach. From a cost-benefit point of view, it may be the case that the areas of highest risk may not coincide with areas that have visible levee deterioration. A more detailed and site-specific analysis would provide not only a more accurate estimate of risk, but also a better mechanism for prioritizing levee maintenance and repairs.

SHANNON & WILSON, INC.

_____
Hollie L. Ellis
Vice President
Director of Computer Sciences

_____
Gregory R. Fischer, P.E.
Vice President

HLE:GRF/hle

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TABLE 1
GREEN RIVER VALLEY FLOOD PHASES

Phase	Flow at Auburn Gage	Description	Condition
1	5,000	Internal alert	
2	7,000	Minor flooding	Lowland flooding in valley upstream of Auburn
3	9,000	Moderate flooding	Flooding of varied depths occur in valley upstream of Auburn and lower Mill Creek basin. SE Green Valley Rd and West Valley Rd may overtop.
4	12,000	Extreme flooding	Levees may exhibit seepage and/or weaken from saturation

(DNR, 2001)

TABLE 2
HISTORIC FLOOD PEAKS
AUBURN PROVISIONAL DATA

Date	Flow (cfs)
November, 1990	11,500
November, 1995	11,200
February, 1996	12,400
December 30, 1998	9,580
November 26, 1999	9,050
December 16, 1999	8,990

(DNR, 2001)

TABLE 3
UNDERSEEPAGE PARAMETERS

Parameter	Expected Value	Estimated Low Value	Estimated High Value
Permeability Ratio, kf/kb	550	100	1000
Blanket Thickness, z (ft)	13.5	2	25
Substratum Thickness, d (ft)	40	30	50
Landside Levee Height, h (ft)	5	1	9

Note: Parameters are defined in Figure 6.

TABLE 4
SLOPE STABILITY PARAMETERS

Parameter	Expected Value	Estimated Low Value	Estimated High Value
friction angle (degrees)	33	29	37
cohesion, c (psf)	15	0	30
riverside slope angle (degrees)	1.7H:1V	1.4H:1V	2H:1V
landside slope angle (degrees)	1.8H:1V	1.4H:1V	2.2H:1V
landside levee height (feet)	5	1	9

TABLE 5
GRFCZD STRUCTURE AND CONTENT VALUATION

Type	Estimated Total Improved Value	Calculated Content Value
Residential	$68,600,000	$41,200,000
Non-residential	$3,421,550,000	$1,983,090,000
Total	$3,490,150,000	$2,024,290,000

TABLE 6
SUMMARY OF HEC-FDA RESULTS

Nominal P_f [1]	Stage-Discharge Uncertainty, ft	
	2.5	2.0
0.35	$65,730,000	$59,300,000
0.25	$49,000,000	NR[2]
0.05	$15,500,000	NR

(1) Probability of levee failure from 1 to 10 feet below levee crest
(2) Not run

LEGEND

〜〜 Water Feature
—— Green River
—— Flood Control Zone District
—— Basin Boundary

KING COUNTY

Detailed Map Area

N

0   2   4

Scale in Miles

NOTE

Figure based on drawing provided
by King County Water and Land
Resources Division.

Green River Flood Control Zone District
Risk Analysis
King County, Washington

**GREEN RIVER BASIN
MAP VIEW**

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SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants   **FIG. 1**

King County Flood Phase Definitions

Minor flooding

Moderate flooding

Extreme flooding

USACE definition of flood stage

Target maximum flow from Howard A. Hanson Dam operations

STAGE, ft
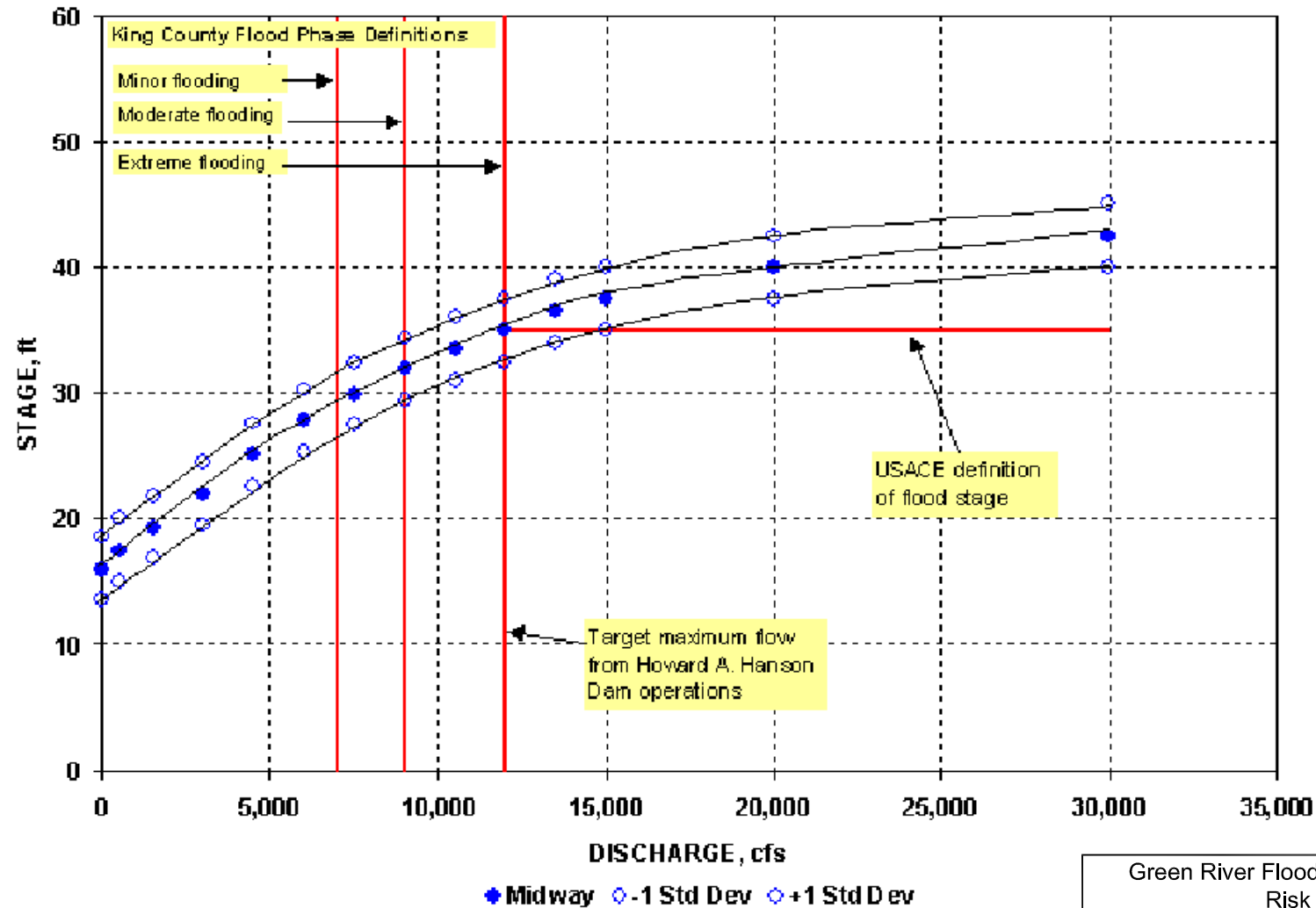
DISCHARGE, cfs

◆ Midway   ◇ -1 Std Dev   ◇ +1 Std Dev

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Risk Analysis
King County, Washington

**STAGE DISCHARGE
RATING CURVE
RM 19.5 BELOW MIDWAY CREEK**

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FIG. 2

FIG. 2
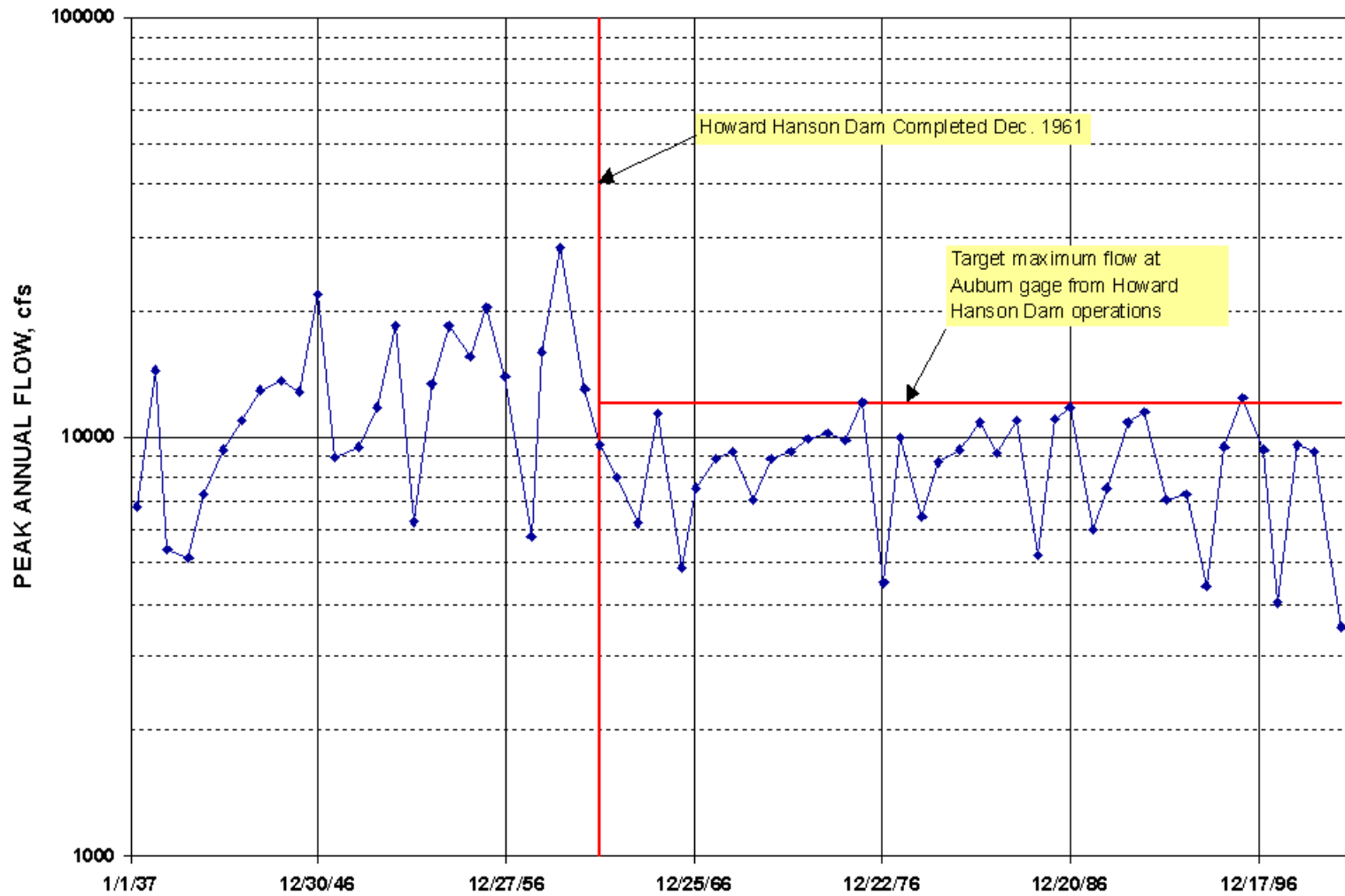
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Risk Analysis
King County, Washington

**PEAK ANNUAL FLOWS
AUBURN GAGE**

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FIG. 3

FIG. 3

PEAK FLOW, TUKWILA GAGE (y-axis)

PEAK FLOW, AUBURN GAGE (x-axis)

◆ Peak Flow ——Linear Least Squares Fit

Green River Flood Control Zone District
Risk Analysis
King County, Washington

**PEAK FLOW CORRELATION
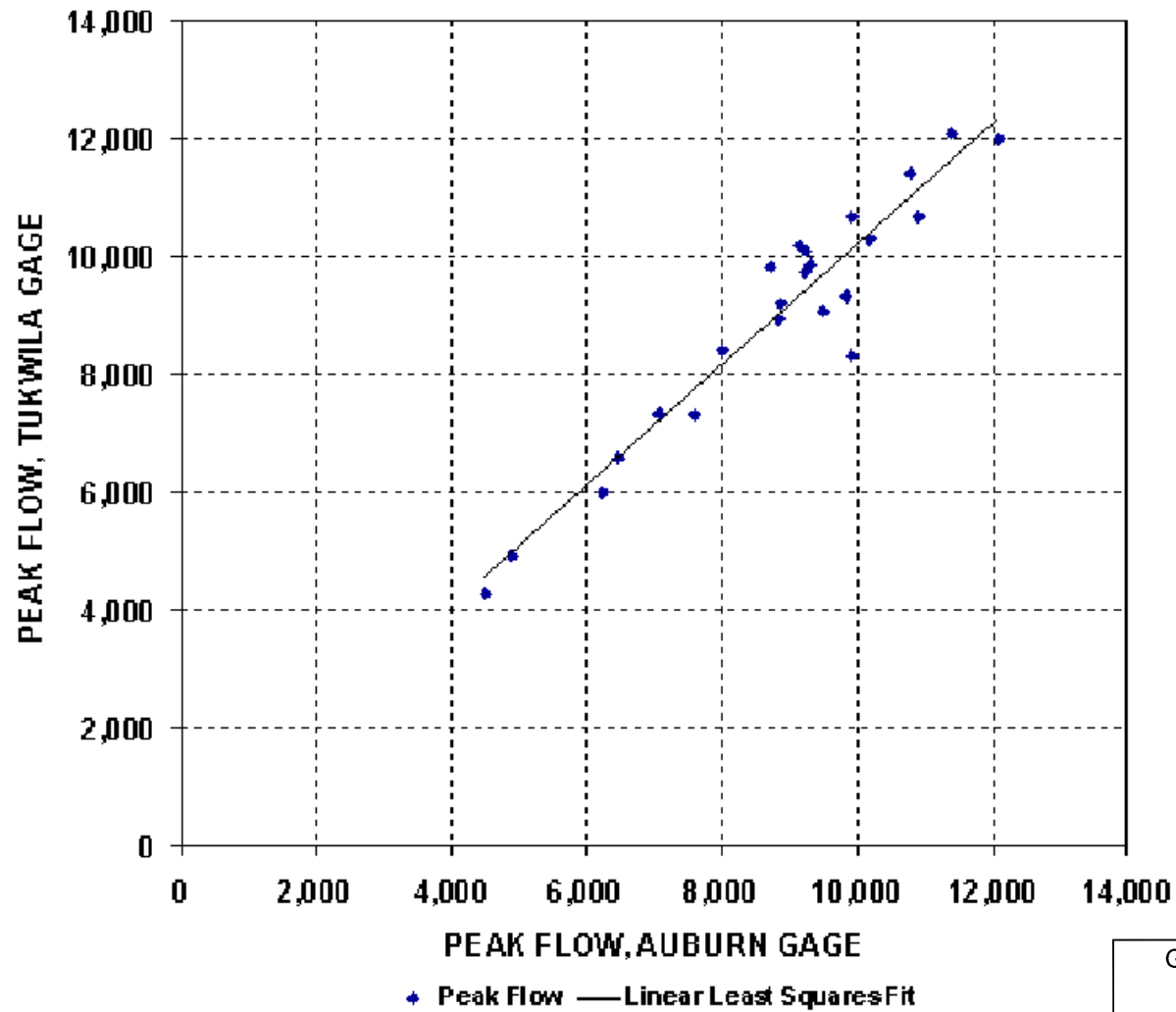AUBURN AND TUKWILA GAGES**

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Geotechnical and Environmental Consultants   **FIG. 3A**

FIG. 3A

Target maximum flow at Auburn gage from Howard A. Hanson Dam operations

Legend: - -△- - 95%   —△— Post-Dam   —▲— 5%   —□— Pre-Dam

X-axis: EXCEEDANCE PROBABILITY
Y-axis: EXPECTED PEAK DISCHARGE, cfs
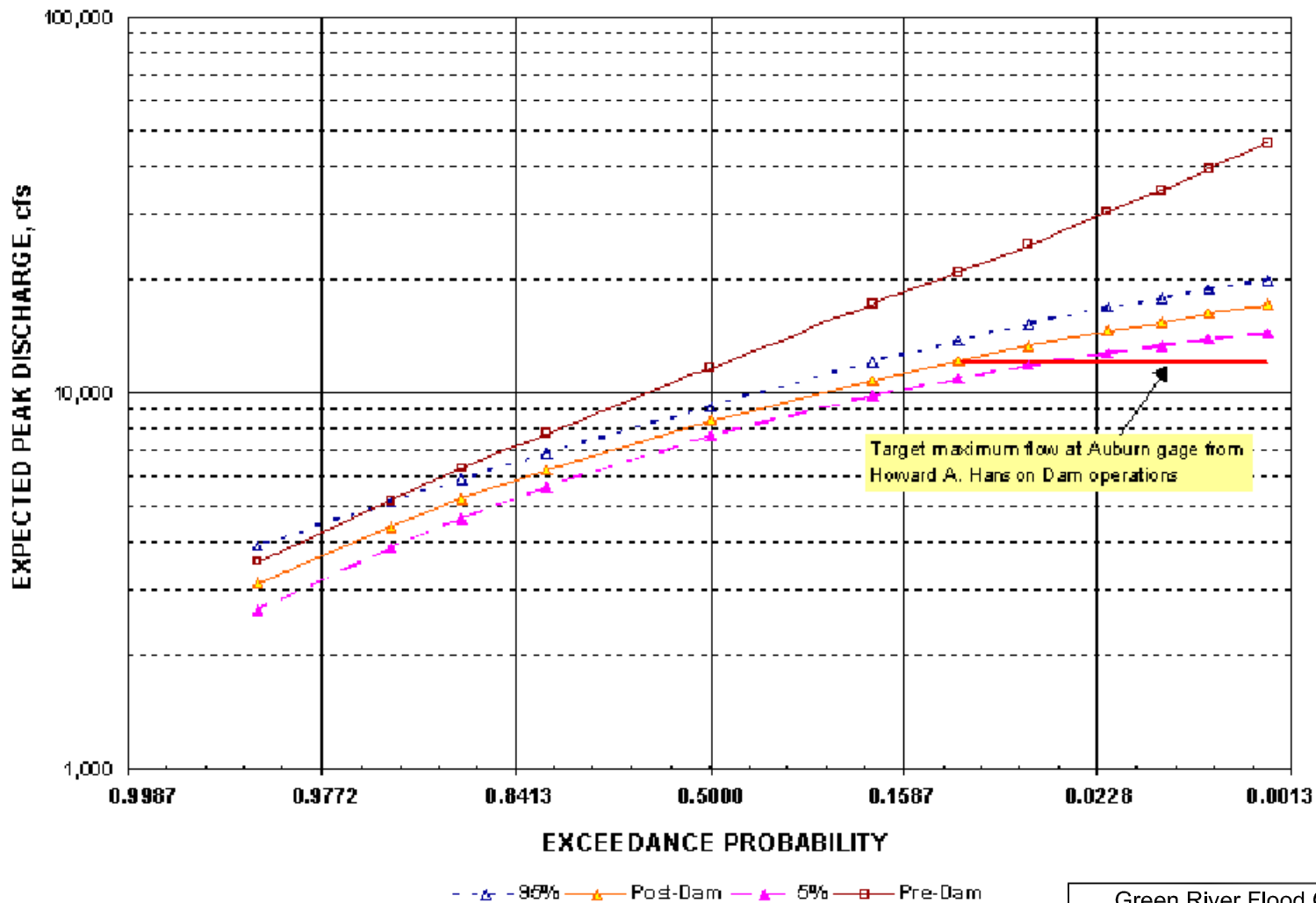
Green River Flood Control Zone District
Risk Analysis
King County, Washington

**EXCEEDANCE PROBABILITY
DISCHARGE CURVE
AUBURN GAGE**

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FIG. 4

FIG. 4

Target maximum flow at Auburn gage from Howard A. Hanson Dam operations

RETURN PERIOD, years

EXPECTED PEAK DISCHARGE, cfs

··△·· 95%   —△— Post-Dam   — △ — 5%   —□— Pre-Dam

Green River Flood Control Zone District
Risk Analysis
King County, Washington

**RETURN PERIOD
DISCHARGE CURVE
AUBURN GAGE**

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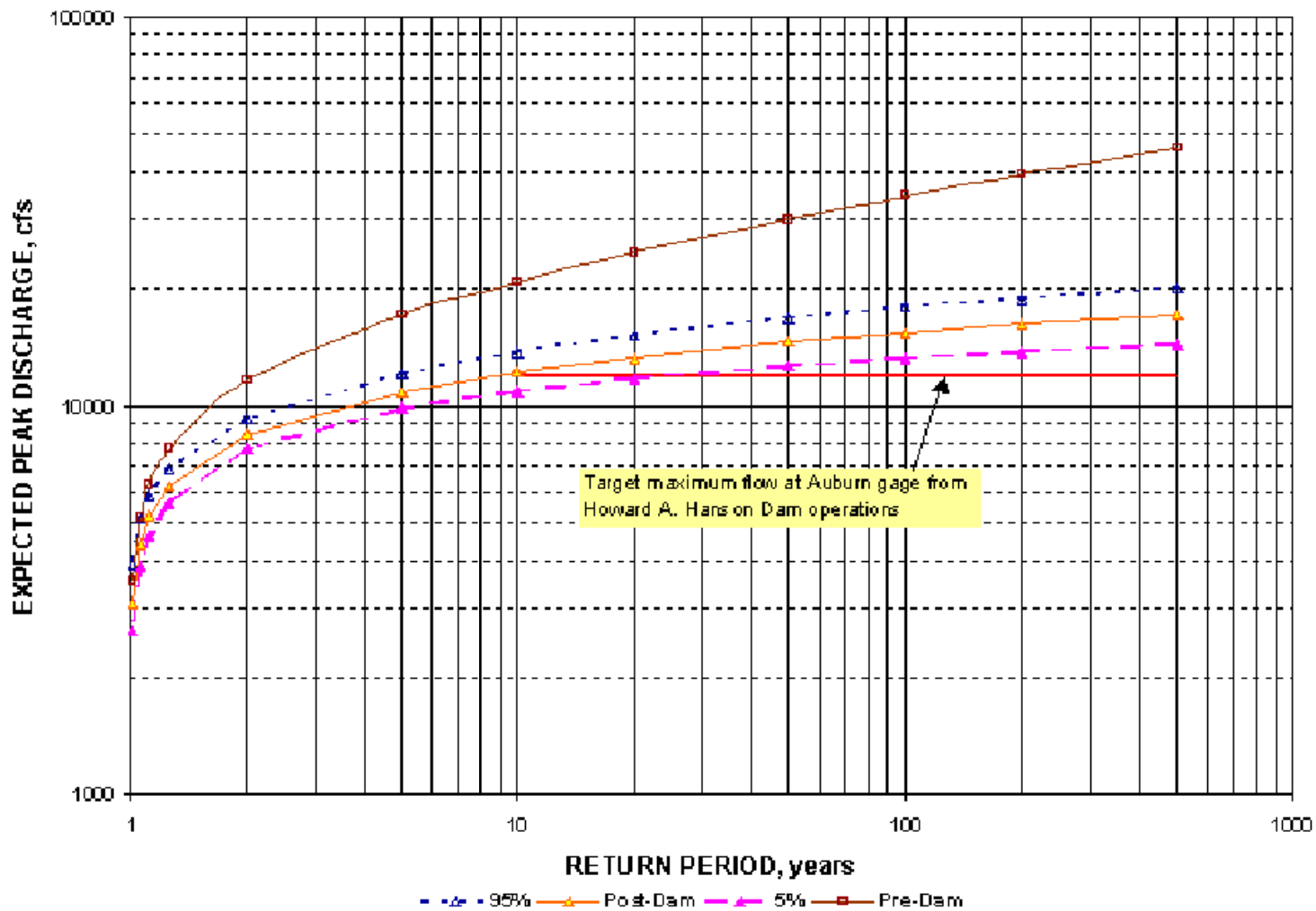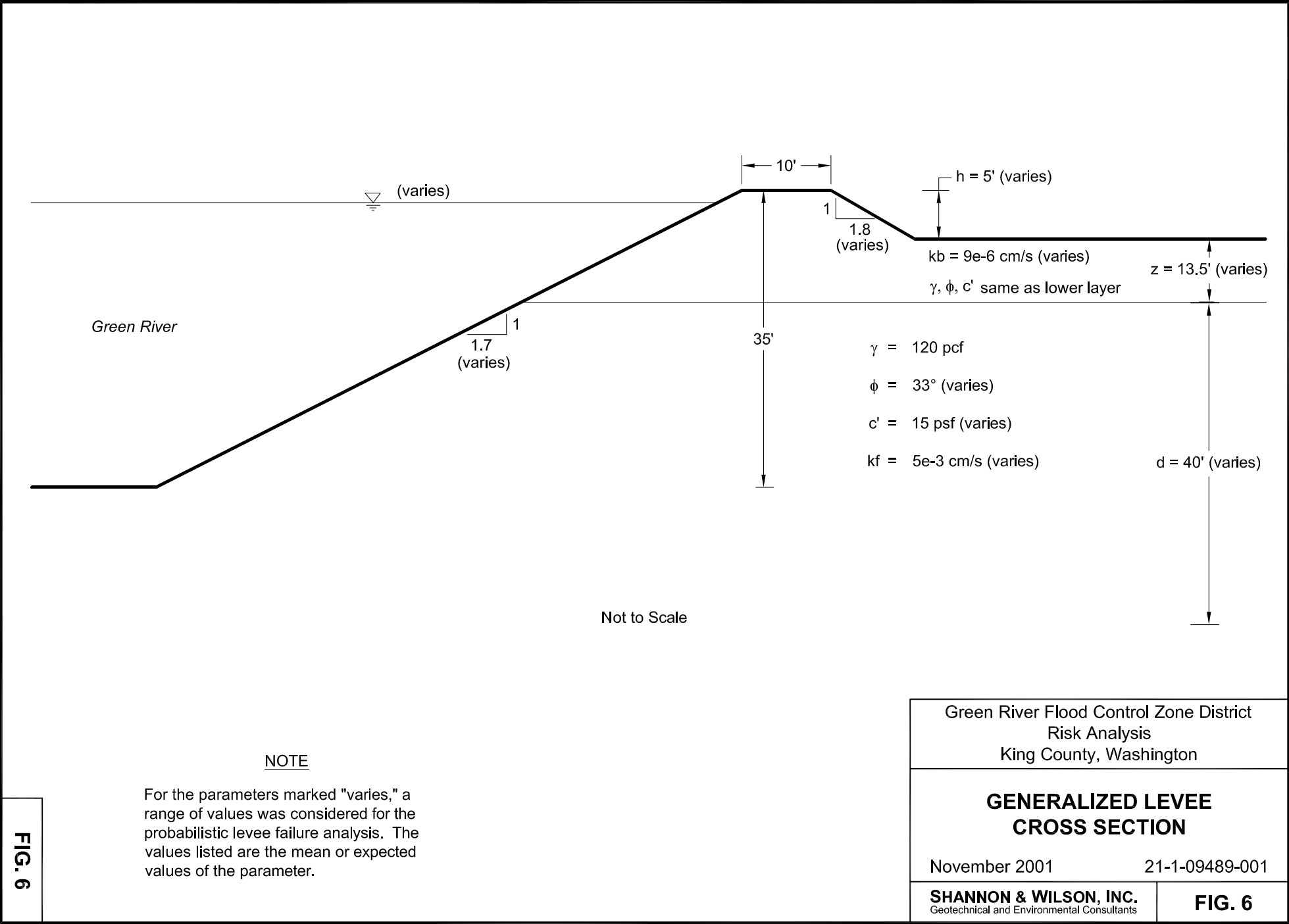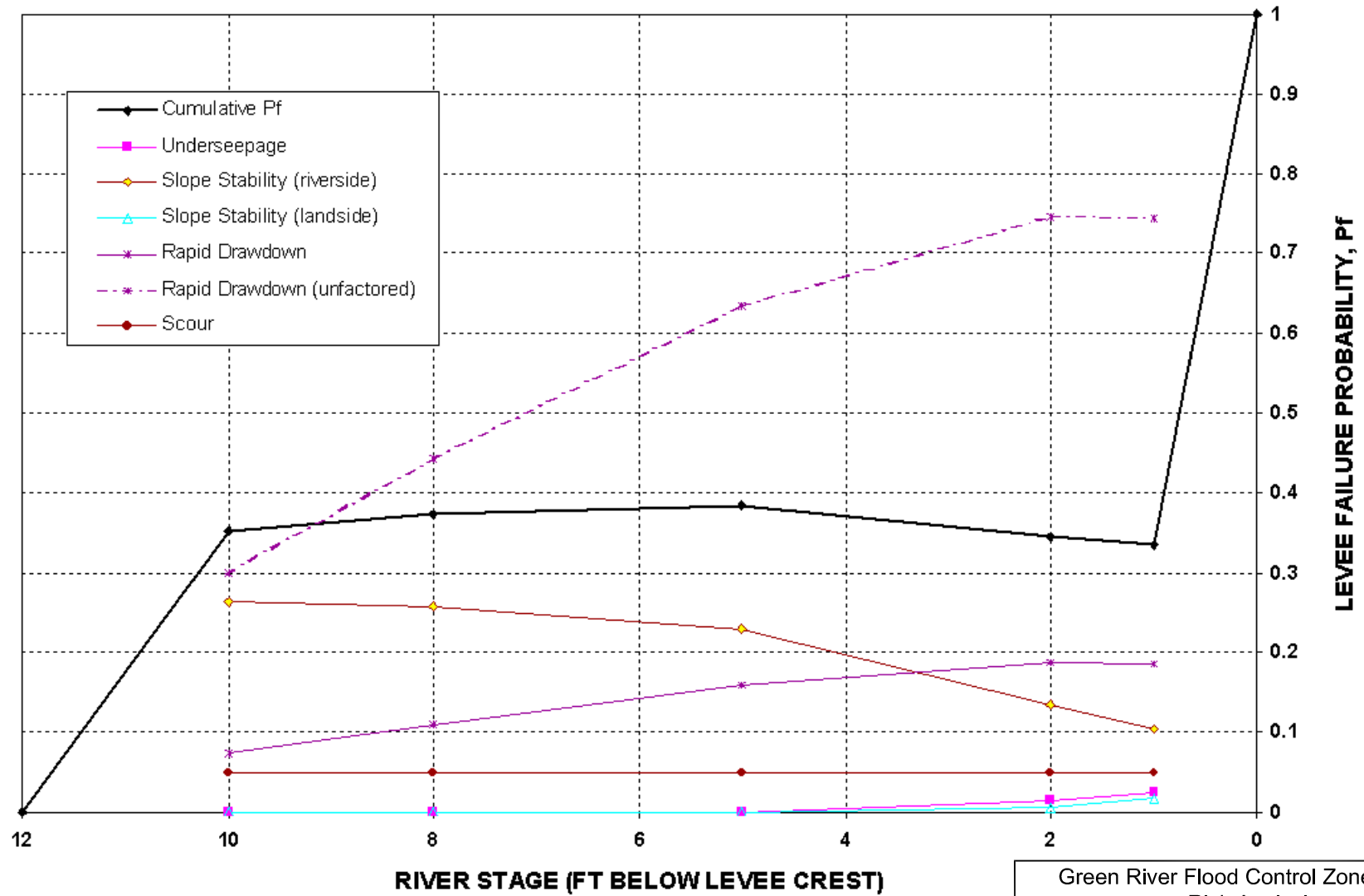
FIG. 5

FIG. 5

10'

h = 5' (varies)

(varies)

1
1.8
(varies)

kb = 9e-6 cm/s (varies)

z = 13.5' (varies)

γ, ϕ, c' same as lower layer

Green River

1
1.7
(varies)

35'

γ = 120 pcf

ϕ = 33° (varies)

c' = 15 psf (varies)

kf = 5e-3 cm/s (varies)

d = 40' (varies)

Not to Scale

NOTE

For the parameters marked "varies," a range of values was considered for the probabilistic levee failure analysis. The values listed are the mean or expected values of the parameter.

Green River Flood Control Zone District
Risk Analysis
King County, Washington

**GENERALIZED LEVEE
CROSS SECTION**

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FIG. 6

FIG. 6

RIVER STAGE (FT BELOW LEVEE CREST)

LEVEE FAILURE PROBABILITY, Pf

Legend:
- Cumulative Pf
- Underseepage
- Slope Stability (riverside)
- Slope Stability (landside)
- Rapid Drawdown
- Rapid Drawdown (unfactored)
- Scour

Green River Flood Control Zone District
Risk Analysis
King County, Washington

**LEVEE FAILURE
PROBABILITY FUNCTION**

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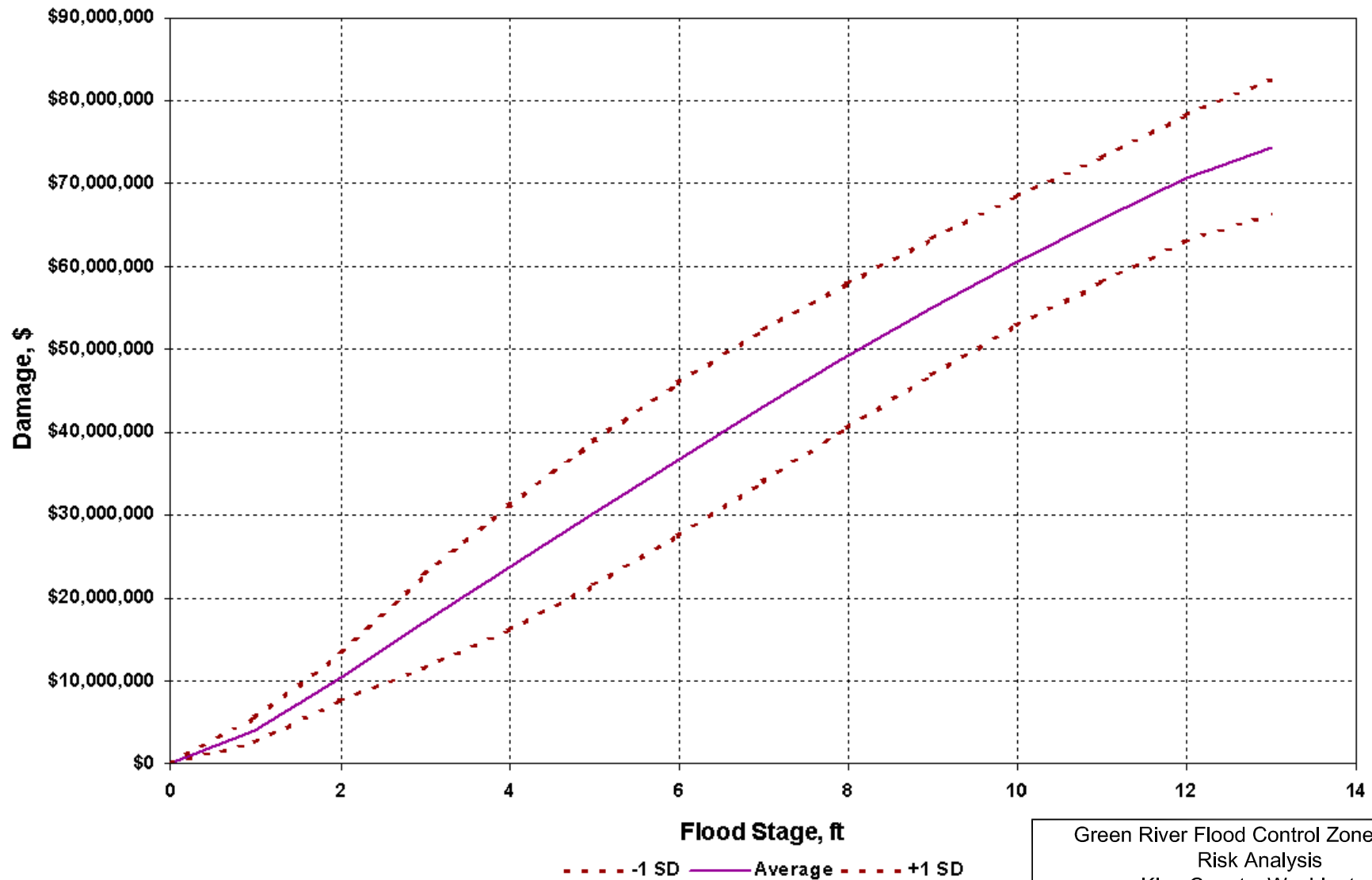Geotechnical and Environmental Consultants

FIG. 7

FIG. 7

Flood Stage, ft

- - - - -1 SD ——— Average - - - - +1 SD

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Risk Analysis
King County, Washington

**RESIDENTIAL STAGE
DAMAGE FUNCTION**

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FIG. 8

FIG. 8

Flood Stage, ft

- - - - -1 SD ——Average - - - +1 SD

Green River Flood Control Zone District
Risk Analysis
King County, Washington

**NON-RESIDENTIAL STAGE
DAMAGE FUNCTION**

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Geotechnical and Environmental Consultants
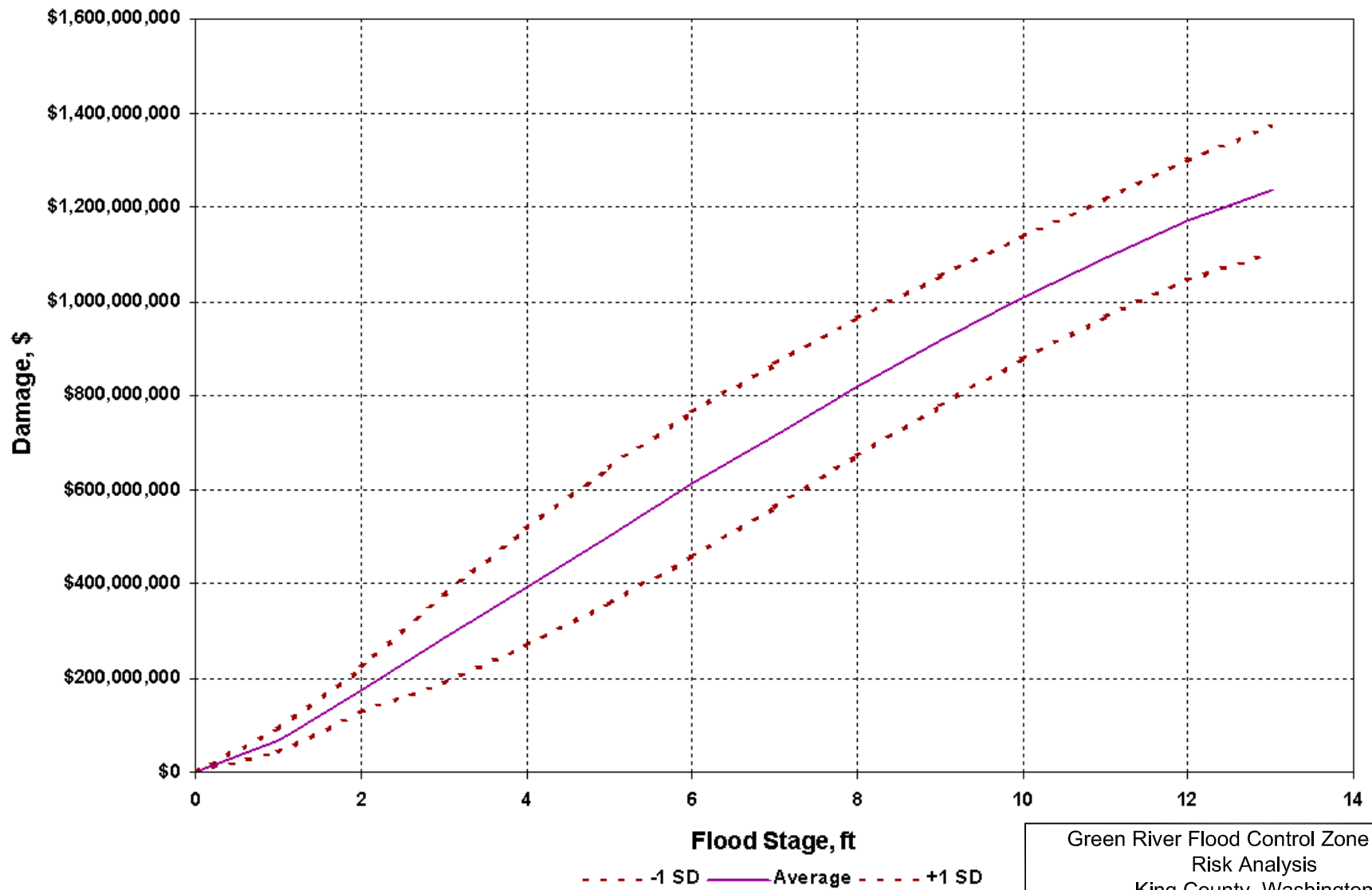
FIG. 9

FIG. 9

APPENDIX

**IMPORTANT INFORMATION ABOUT YOUR
GEOTECHNICAL REPORT**

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Proposal

Date: November 1, 2001
To: Mr. Dave Clark
Department of Natural Resources
King County, WA

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the

consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

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The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

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